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Testing and Evaluation of Irvine Creek Bridge

N.J. Patel
Research Engineer
Structural Research Group

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The Editor, Technical Publications
Ministry of Transportation and Communications
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Canada M3M 1J8
(416) 248-7226

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Abstract:

This report describes the structural evaluation and live load test on a structural steel truss bridge called Irvine Creek Bridge, located in the village of Elora near Fergus. The structure is a two-lane bridge with a sidewalk and two identical steel trusses spanning 31.7 m over Irvine Creek. An earlier evaluation report prepared by consulting engineers suggested that the floor beams and the trusses were under strength and not capable of carrying OHBDC loads without posting. The load test performed on the structure, however, revealed that all components of the bridge superstructure (including deck, stringers, floor beams and steel trusses) have the reserve capacity to carry the full OHBDC live loads without posting. However, it is recommended that the bridge be repaired immediately to remedy some of the defects cited by the consultant's report which are also described in this report.

Key Words: bridge strength, load testing, OHBDC load, Irvine Creek Bridge, reserve capacity, steel truss bridge

Comments:

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1/ INTRODUCTION

Irvine Creek Bridge, MTC Site No. 35-185, is located on Woolwich street in the Village of Elora, Town of Salem, near Fergus in the County of Wellington, District of Stratford. The structure carries north- and southbound traffic along Woolwich street on County Road 18 and crosses Irvine Creek with 22.5° skew.

Details of the Irvine Creek Bridge are shown in Figures 1, 2 and 3. The bridge has a single span, and two Warren-type pony trusses with a reinforced concrete deck slab supported on steel stringers and floor beams. The substructure is of reinforced concrete construction.

On behalf of the County of Wellington, an investigation and evaluation report on the bridge was prepared in July, 1982, by McCormick, Rankin & Associates Ltd., Consulting Engineers, Mississauga. They recommended several repairs including replacement of the bridge deck and load restriction on the bridge. Subsequent to that, the Structural Office of MTC requested the Research and Development Branch to load test the bridge to determine its actual load carrying capacity.

This report contains the results and recommendations from the load test performed on the above structure on July 25, 1984.

2/ DESCRIPTION OF BRIDGE

As previously mentioned, the Irvine Creek Bridge is a single span, Warren-type, steel pony truss structure crossing Irvine Creek with 22.5° skew. The structure was designed and built in 1952 for an H20 truck loading.

The steel trusses span 31.7 m and clear width between curbs is 7.32 m, which accommodates two lanes of traffic. The structure also accommodates a 1.80 m cantilevered concrete sidewalk. Details of the bridge are shown in Figures 1, 2 and 3. The bridge deck consists of a 190 mm concrete slab, supported by six stringers (S380 x 64), spaced at 1346 mm centres which, in turn, are supported by eight floor beams (W760 x 161) spaced at ±4.034 m centres. The sidewalk is a 101 mm thick concrete deck supported on cantilevered steel brackets off the west truss.

Although the original drawing of the deck does not indicate their use, the structure now has approximately 50 mm of asphalt topping and the associated waterproofing membrane, except for the sidewalk.

The stringers are simply supported over floor beams located at each panel point of the truss. The truss consists of rolled steel members, except the top chord which has built-up members fabricated using steel channels and plates and riveted connections. The truss has fixed bearings on the south end of the structure and free bearings on the north end.

The investigation and evaluation of Irvine Creek Bridge [1] conducted by McCormick, Rankin & Associates Ltd., identified the following problems on the structure:

- severe scaling of the top surface of the deck;
- significant cracking and delamination of the under side of the deck slab;
- active corrosion of reinforcing steel within the concrete deck slab;
- a 90 mm hole at the south 'fixed joint' due to spalling and crumbling of concrete, and curbs in poor condition;

- a leaking and deteriorated deck expansion joint permitting water, salt and debris to fall into the floor beam and bearing seat below the joint; and
- rusted structural steel components due to lack of maintenance painting over the years.

The concrete is air-entrained, but the air content in the cement matrix is unevenly distributed and irregularly shaped. Because of these characteristics, the concrete is inadequately protected from cyclic freezing and thawing. However, laboratory testing shows the compressive strength of the concrete to be relatively high, with values ranging between 35 MPa and 68.1 MPa.

3/ BRIDGE EVALUATION

The evaluation of the bridge superstructure, with the aid of computer analysis, was performed prior to testing the bridge, according to the Ontario Highway Bridge Design Code [2]. The capacity of the deck slab (calculated using the simplified method given in Section 14 of the Code) is approximately 750 kN, as opposed to the minimum of 420 kN required by the Code for unposted bridge decks. In evaluating the deck slab load capacity, the lower concrete strength of 35 MPa is used as per the condition survey report by Golder Associates and the deck is considered to be of non-composite construction. In the absence of original deck drawings, the minimum amount of reinforcing steel is assumed for the purpose of deck evaluation.

The stringer dead load and superimposed dead load moments were calculated by using the simple beam analogy. The live load moments on stringers were calculated using grillage analysis [3]. The floor beam moments were obtained by using the stringer reactions transferred onto the floor beams for various critical loadings. A dynamic load allowance of 0.3, the minimum required for floor beams by the OHBDC, was used to obtain the factored live load effects. These maximum moments for stringers and floor beams, derived as described above, are given in Table 1. It is evident from the table that the stringers have adequate strength to sustain full OHBDC loads while the floor beams are somewhat under-strength to carry the OHBDC live loads and necessitate posting of the bridge.

The axial forces for selected truss members calculated for dead, superimposed dead and live loads are given in Table 2. A comparison of applied factored loads on the members to their respective factored member resistances calculated as per the OHBDC is given in Table 3. A dynamic load allowance of 0.40 was calculated and included in the evaluation of the factored live loads on the truss members. The top chord and some of the bottom chord members were found to have substantially lower strength than required for full OHBDC loads. The web members for the most part have adequate capacity to carry the OHBDC loads except for few members which appear to have strength marginally lower than required.

Since the truss members generally consist of composites of rolled beams, channels and/or plates riveted together, the fact that they are under-capacity may not be critical and they may sustain forces well above their evaluated capacities. The longitudinal bucking load capacity of the truss was also evaluated using the computer program developed by Bakht and Csagoly [4]. The single truck load capacity given by this program is approximately 980 kN per truss. The total OHBDC truck load imposed on a truss at the ultimate limit state using a dynamic load allowance of 0.40 is approximately 1372 kN. This means that the longitudinal buckling capacity of the truss is nearly 71% of that required to sustain the ultimate OHBDC loads.

4/ INSTRUMENTATION AND LOAD TESTING

4.1/ Instrumentation

The proof load test is generally carried out to establish the safe load carrying capacity of a bridge. During the test, the structure is subjected to exceptionally high static loads. The loads are such that responses to them are equivalent to or higher than those caused by the maximum allowable loads on the bridge, including the appropriate values of the load factors and dynamic load allowance. Therefore, to avoid overloading and to study the behaviour of various members under applied loadings, the structure is instrumented extensively and monitored closely during the test.

Irvine Creek Bridge was instrumented using both weldable and demountable type uniaxial electrical strain gauges and linear displacement transducers to monitor stringers, floor beams and trusses during the load testing. The locations of these instruments are shown in Figures 4 and 5 and are described as follows:

1/ Twenty-eight strain gauges were used to measure strains developed in the west truss members (Figure 4). The number of strain gauges attached varied depending upon the critical nature of the member evaluated. For example, a greater number of strain gauges were placed on the west truss because it was considered more critical than the east truss. This is because the west truss is supporting the extra loads of a cantilevered pedestrian sidewalk. Similarly, premature buckling of the cover plate can cause failure of top compression chord members. To monitor such behaviour, the most critical members of the top chord were instrumented with additional strain gauges at the centre point of the member. The east truss was instrumented with 13 strain gauges (also shown in Figure 4). The gauges monitored the axial strains, and hence the axial forces in the truss members under various predetermined, increasing load levels on the bridge.

2/ One strain gauge was fixed to each of the six stringers of the second panel at the midspan location. Similarly, four interior floor beams were instrumented at midspan locations as shown in Figure 5.

3/ Four linear vertical displacement transducers with an accuracy of ±0.05 mm were installed on the west truss and only one transducer was placed on the east truss, as shown in Figure 5. These vertical transducers were attached to the selected stringers, floor beams and panel points of each truss so as to monitor the maximum anticipated vertical deflection in those members under the test loads.

Transverse displacement transducers were also installed to monitor the horizontal movements at both fixed and free bearings of the trusses (Figure 5).

4.2/ Load Testing

Two MTC test vehicles described by Bakht and Csagoly [5] were used for vehicular test loading. Figure 6, and Tables 4a and 4b indicate the various load levels and test truck axle configurations. Figure 7 shows the four transverse locations (lines) of the test vehicles and four longitudinal positions on the structure. The trucks were used either individually or together with a gross weight of each test truck up to 780 kN and then a single truck was used with a maximum total gross weight of up to 890 kN. Two trucks both loaded to 890 kN each were not used to test the bridge because it was thought that the probability of two heavily loaded trucks passing the bridge simultaneously causing a worse load effect is negligible. It has been shown by Holowka [8], based on the longitudinal moment effects on simple spans of 24.4 m and 31.4 m, that the simultaneous loading of two test trucks weighing 890 kN each is equivalent to the factored OHBDC truck loads to within 1%. The case studied, for the slightly lower truck loads of 780 kN each, may not induce the member forces to the levels of the ultimate OHBDC forces on all members; however, the applied loading is considered to be realistic for this particular bridge.

A number of procedures were followed to ensure adequate safety protection. This involved:

- vehicular traffic control by an experienced patrolman,
- continuous monitoring of critical deflection transducers and strain gauges during the loading to catch unexpected large deflections and member strains, and
- incrementing the initial test load (approximately 60% of final load) gradually, providing the structure showed elastic behaviour for the previous load level.

The data from the strain gauges and deflection transducers were recorded with a computer based multi-channel data acquisition system. During the test, none of the strains recorded on any of the gauges monitored were unduly high and all were recoverable.

5/ TEST RESULTS AND DISCUSSION

The bridge structure carried the applied test truck loads without showing visible signs of distress. This observation in itself suggests that the bridge can carry the OHBDC live loads without posting. A final analysis of the strain and deflection measurements from the tests was done, however, to identify possible signs of weakness in the structural elements that remained undetected by visual observation. Relevant graphs are plotted and shown elsewhere in this report to compare and confirm the test results in relation to the detailed analysis conducted prior to the bridge testing for the same test loads. This investigation is divided into the following categories:

- stringers,
- floor beams, and
- truss members.

5.1/ Stringers

Typical strain diagrams for stringers are plotted using the strains measured during the load test and are shown in Figure 8. Only the bottom flange strains are shown in the diagrams since the top flange was not instrumented with strain gauges.

The theoretical stringer moments were calculated using grillage analysis of the deck under applied loading. The corresponding test moments were calculated using the measured strains with a) composite and b) non-composite section properties of the stringers and are shown plotted with stringer locations in Figures 9 and 10. Figure 9 illustrates the loading cases of the trucks weighing approximately 780 kN while Figure 10 illustrates that of a truck loaded to approximately 890 kN. For the grillage analysis of the deck, only the non-composite deck properties were used.

The figures show that the shape of load distribution graphs obtained using the measured strains are generally similar to that calculated with the grid analogy. However, the graphs for theoretical moment values lie between the graphs of the upper bound and the lower bound moments calculated using the fully composite and non-composite section properties

respectively. This observation in itself indicates that the stringer sections are acting partially composite with the concrete deck. The bridge evaluation shows that the stringers have adequate capacity to sustain the OHBDC loads and the test results suggest that the stringers act as a partially composite beams. From these observations, it is concluded that the stringers have strength well in excess of that required by the OHBDC.

5.2/ Floor Beams

The strains measured on the floor beams during the bridge test are plotted in Figure 11 for loading cases with truck loads of 780.0 kN and 890.0 kN. Only bottom flange strains are shown since the top flange was not instrumented.

The moments on the floor beams calculated using the stringer reactions obtained from the grid analysis are given relative to respective moments derived from the strain measurements and floor beam non-composite and composite section properties in Table 5. The load transferred to the floor beams is usually obtained without allowing for the possible load sharing effects developed by the longitudinal stiffness of the stringers, the concrete deck, and/or floor beam flexibility. A technique has been developed by Bakht [6], which gives correction factors for moments on the floor beams. A simplified method is given to account for the load sharing effect between the adjacent floor beams. Table 5 also includes the corrected value of the floor beam moments due to load sharing effect of the floor beams.

For all the floor beams, the values thus obtained compare very well with the measured composite section moments for each of the loading cases. Under the applied test truck loadings, the floor beams behaved linear elastically and the measured composite section moments were about 75% of the theoretically calculated moments. These calculated moments were not corrected for load sharing as is usual in the design analysis and evaluation of the floor beams, but when corrected calculated moments were used, they compared very closely to the measured composite moments. At this point, it should be noted that even though the steel floor beams have been built to be non-composite with the concrete deck, as shown in the

original drawings, they have been found to act compositely with it since they have carried larger applied loads than the naked steel could have carried.

Furthermore, the sum of all the floor beam moments at the midspan due to any loading was calculated and compared with the sum of the corresponding moments calculated from the measured strains using the composite section modulus. It was found that the sum of measured moments was about 75% of the sum of theoretically calculated moments.

This confirms that the floor beams have the reserve capacity to take greater live loads than those applied by the test trucks and therefore can safely carry the OHBDC live loads without posting.

5.3/ Truss Members

Two steel pony trusses support the roadway deck and sidewalk area. They are determinate type trusses and hence, with the exception of the vertical members, require all the members to actively participate in transferring the applied loads to the end supports. The vertical members provide lateral restraint to the top chord members so failure of one member can prove disastrous to the bridge structure. Consequently, care was taken not to overload the bridge while load testing by constantly monitoring the behaviour of various members under applied loading and then comparing the test strains with the anticipated strains obtained by the bridge analysis conducted prior to bridge testing. No mishaps occured and no abnormal strains and deflections were recorded during the test. The testing was completed satisfactorily within six hours.

The components of the built-up truss members have shown some strain variations under various loading cases as illustrated in Figure 12. In the channels of the middle section of the inclined end chord member, strain was found to be higher than on the plate section. However, the middle sections of other top chord members generally showed higher strain in the plate sections than the channel sections. The bottom chord members were of rolled beam sections and, in general, the inner flange of the section showed higher strain than the outer flange, though the difference is not significant. The web members were also the rolled beam type and were not

instrumented on either of the flanges, since it was assumed that they would exhibit approximately equal load distribution under various test loadings. Accordingly, only the web of the rolled beam was instrumented.

Though variations in strains were observed in the elements of the top chord built-up sections, the difference in variation was not significant enough to warrant an adjustment in calculating the actual forces induced in the top-chord members under the test truck loads. Thus, for the selected top chord, bottom chord and web members, the member forces were calculated for the applied truck loadings with the assumption that all the joints of the trusses were pin-connected joints with one end support on the free bearing. Similarly, the actual member forces were obtained from measured strains for each applied load case. The following bridge loading cases were used to prepare Tables 6a and 6b:

- two trucks each weighing 780 kN, and
- a single truck with total load of 890 kN.

The measured top chord member forces were found to be lower than calculated member forces for the most part; measured values ranged between 80 to 95% of the calculated values.

The magnitudes of the measured forces of the bottom chord were very much lower than the corresponding calculated member forces. The measured member forces were usually below 50% of the calculated values. The bottom chord nearest to the bearings generally showed a compressive force, while the calculated force was obviously tensile. It is thought that the observed reduction in forces in the bottom chord and/or the compressive forces exhibited on the bottom chord nearest to the bearings may be due to the following:

1/ the tensile forces transferred to the bottom chord are being shared by stringers and the concrete deck and/or

2/ the free bearing movements are becoming restricted by rusting and accumulating dirt and debris.

To further determine the cause of the above finding, an analysis of the truss was performed assuming fixed bearings on both ends of the truss and

the structure loaded with two trucks weighing 780 kN each. This was done using the STRUDL program. The results are given in Table 7 with the fixed bearings values shown in parentheses.

Comparison of these forces in members of the truss revealed that the top chord and web members are obviously unaffected, but the bottom chord forces are greatly altered. The bottom chord nearest to the bearings is now required to sustain significant compressive force. However, the measured compressive force is very much lower than the theoretical values. On the other hand, if the bearings are actually fully fixed as assumed, it is also possible that part of the compressive load is shared by the deck and stringers.

Surprisingly, for the rest of the bottom chord members, it was found that the forces calculated by assuming fixed bearings are closer to the measured values. In general, the measured values are smaller by approximately 0 to 20% than the analytical values with the fixed bearings. Nevertheless, it was not possible to carry out a repeat test with bearings tested to be functionally operational due to a tight testing schedule and financial constraints. It is concluded that the observed reductions in bottom chord forces are a combined effect of:

- load sharing between bottom chord, stringers and or/deck, and
- the effect of semi-rigid bearings.

It is suggested that in a future test of a similar bridge, the effects of semi-rigid and free bearings be studied closely.

The web members had measured member forces between 65 to 75% of calculated member forces, except for one compression member (5-6) on which the measured member forces were between 0 to 30% higher than calculated member forces.

The vertical displacements of the truss measured at nodal point nine by two trucks, each loaded to 780 kN are given relative to the deflections calculated assuming free bearings in Table 8. Also, the relative deflections for load level 4 are shown in parentheses, assuming fixed bearings at both ends of the truss. The measured vertical deflections were found to be usually more than 50% less than calculated values for

free bearings and about 20 to 35% lower than calculated values for fixed bearings. The calculated deflection assuming free bearing is generally greater than the measured deflection for the following reasons:

- The bearings become semi-rigid or rigid after years of service and inadequate maintenance.
- The bridge/truss acting as a structural system has considerably higher rigidity than usually modelled theoretically.

Some movement was recorded at the fixed end of the bridge, ranging from 0 to 12 mm. The free end bearing at the north end of the bridge was recorded moving horizontally from between 3 mm to 19 mm when the structure was loaded with two trucks weighing 780 kN each.

The truss and its members, however, behaved linear elastically under the applied truck loads, which could have caused force effects quite close to OHBDC level 3 loads. The stresses recorded in the test confirm that the truss elements have considerably greater capacity than required to sustain the test truck loads and hence the OHBDC loads without posting.

6/ CONCLUSIONS AND RECOMMENDATIONS

6.1/ Evaluation as per OHBDC

The following are the conclusions drawn from the bridge evaluation study carried out as per the OHBDC.

1/ The strength capacity of the bridge deck slab is found to be in excess of sustaining OHBDC live loads.

2/ The evaluated strength values of the stringers are greater than factored OHBDC loads.

3/ The non-composite section resistance of the floor beams is marginally lower than the strength required by OHBDC.

4/ The truss top and bottom chord members have up to 30% lower member resistances than required to sustain full OHBDC loads.

5/ The web members have sufficient member resistance to carry legal OHBDC loads.

6.1.1/ Load Test Results

1/ The load test performed on the structure showed that the stringer beams and the concrete deck behave as a partially composite section. The theoretical moment values obtained using grillage analysis have a similar shape to and lie between the upper bound and lower bound moment values obtained using the measured strains and with fully composite and non-composite section properties, respectively. The following observations were made:

- The bridge carried quite heavy vehicles and the stringers were well within the elastic stress range.

- The load test revealed that the stringers act as partially composite beams and the evaluation study showed that the stringers, even acting non-compositely, have strength values greater than OHBDC factored loads.

These observations confirmed that the stringer/deck system has sufficient capacity to sustain OHBDC live loads without posting.

2/ The floor beam moments calculated prior to the load test were based on the scheduled test truck loadings at various positions and then compared with the moments obtained by the measured strains on the bottom flange using the composite section modulus. The measured moments were approximately 75% of the uncorrected, theoretically calculated moments which are usually used to design/evaluate floor beams. The measured moments using steel sections resulted in values less than 50% of the calculated moments. These theoretically calculated moments were then corrected for the effects of load sharing between floor beams as described by Bakht [6] and again compared with the measured composite moments obtained at the centre of the floor beam span. The measured composite moments compared very well to the corrected calculated moments; they ranged from 91 to 100% of the corrected calculated moments. On the basis of above load test results, the floor beams have adequate reserve capacity to take legal loads of Ontario without posting.

3/ The components of the built-up top chord truss members showed some stress variations under each loading case but the difference in stress variation was not so pronounced as to warrant adjustment in calculating the actual measured forces induced in the top chord members. However, for the bottom cord and web members of the truss, which are rolled beam sections, the distribution was generally found to be uniform.

The measured top chord member forces were usually lower than the theoretically calculated values. The measured values ranged between 80 to 95% of the theoretical values. Similarly, the measured bottom chord member forces were also very much lower than the theoretically calculated values. In fact, the bottom chord nearest to the bearings gave compressive instead of tensile force values. For the rest of the bottom chords, the measured values were generally 50% lower than the theoretical values. An analysis conducted with fixed bearings showed larger compressive force values on the bottom chord nearest to the

bearings than the measured force values and gave smaller tensile force values for the rest of the bottom chord members. However, rather than repeating the test with bearings proven to be functioning normally, it is concluded that the observed reductions in bottom chord forces are a combined effect of:

- load sharing between the bottom chord, stringers and or/deck, and
- defective bearings.

It is suggested that in a future test of a similar bridge, the effect of fixed bearings on the bottom chord forces should be studied closely.

The end web members showed the measured member forces to be quite close to the theoretical member forces. However, towards the middle of the truss the measured forces were about 25% lower than theoretically calculated forces. The load tests indicated that all the truss members were well within the elastic stress range and measured member forces were generally lower than theoretically calculated forces (the degree of reduction depended on the member type). These findings confirm that the truss has a adequate capacity to carry any load which is legal in Ontario.

4/ The measured vertical deflections for truss were more than 50% less than theoretical values for free bearings and about 25% less for fixed bearings. This finding may be attributed to the following:
   - The bearings become semi-rigid after years of service and inadequate maintenance.
   - The bridge/truss acting as a structural system has a higher rigidity than is usually modelled theoretically.

5/ The following recommendations from the consultant for structure repairs should be followed:
   - provide new strip-seal expansion joints,
   - clean, sandblast and paint the superstructure,
   - inspect and repair all connections (since the connections were not tested), and
   - repair bearing seats and abutments.

In future, if replacement of the existing deck is undertaken, the consultant's other recommendations should be followed, namely; modification of the floor beams and stringers to provide composite structural action, waterproofing and paving the deck, and paving the approach to meet with the reconstructed deck surface.

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Table 1a/ Unfactored Moments on Stringers and Floor Beams with OHBDC Live Load

Member	Maximum Moments, kN·m		
	Dead Load	Superimposed Dead Load	Live Load
Stringer	13.8	4.8	98.1
Floor Beam	177.1	57.6	704.4

Table 1b/ Factored Moments and Member Resistance on Stringers and Floor Beams with OHBDC Live Load

Member	Maximum Factored Moments, kN·m	Factored Resistance, kN·m		
		Non-Composite Section		Composite Section
		Laterally Supported	Laterally Unsupported	
Stringer	161.1	236.0	173.1	
Floor Beam	1269.9	1171.6	--	5053

Table 2/ Factored Axial Forces on Selected Members of West Truss with OHBDC Live Load

Member	Factored Axial Forces, kN			
	Dead Load	Superimposed Dead Load	Live Load	Total Force
Top Chord				
1-2	-765.8[1]	-255.2	-1242.1	-2263.1
2-6	-968.9	-323.0	-1589.1	-2881.0
6-10	-1276.0	-425.4	-1944.6	-3646.0
10-14	921.8[2]	-307.2	-1519.2	-2748.2
Bottom Chord				
1-5	567.3	189.1	920.3	1676.7
5-9	1205.2	401.8	1914.0	3521.0
9-13	1181.5	393.8	1895.3	3470.6
Web				
2-5	570.8	190.3	990.6	1751.7
5-6	-404.5	-134.8	-750.6	-1289.8
6-9	251.4	83.8	523.2	858.4
9-10	284.0	94.7	563.2	941.9
2-3	150.0	50.0	463.6	663.6
6-7	150.0	50.0	463.6	663.6

[1] - Compressive
[2] + Tensile

Table 3/ Factored Axial Forces (in Table 2) vs Factored Member Resistance

Member	Member Forces, kN	Member Resistance, kN
Top Chord		
1-2	-2263.1	-1967
2-6	-2881.0	-2046
6-10	-3646.0	-2877
10-14	-2748.2	-2046
Bottom Chord		
1-5	1676.7	1804
5-9	3521.0	2576
9-13	3470.6	2576
Web		
2-5	1751.7	1498
5-6	-1289.8	-1562
6-9	858.8	1498
9-10	941.9	1498
2-3	663.6	1084
6-7	663.6	2202

- Compression
+ Tensile

Table 4a/ Tandem Weight of Testing Vehicle, Vehicle No. 1 (Kenworth) Loading

Load Level	Total Load (kN)	Number of Blocks	Axle Weight, kN		
			Front Axle	Front Tandem Each Axle	Rear Tandem Each Axle
1	442.6	24	47.6	92.5	105.0
2	552.9	36	49.4	115.2	136.6
3	665.4	48	51.6	138.3	168.6
4	777.6	60	53.4	161.5	200.6
5	890.0	72	56.0	185.2	231.8

Table 4b/ Tandem Weight of Testing Vehicle, Vehicle No. 2 (Mack) Loading

Load Level	Total Load (kN)	Number of Blocks	Axle Weight, kN		
			Front Axle	Front Tandem Each Axle	Rear Tandem Each Axle
1	432.4	24	43.6	89.9	104.5
2	545.3	36	44.5	113.0	137.4
3	657.0	48	45.8	135.7	169.9
4	770.0	60	46.7	158.8	202.8
5	882.0	72	47.6	181.0	236.2

Table 5/ Calculated Floor Beam Moments Relative to Measured Floor Beam Moments for Various Loading Cases

Loading Case	FLOOR BEAM MIDSPAN MOMENTS, kN•m				
	Calculated Max. Floor Beam Moment at Midspan	Corrected Moment for Load Sharing	Measured Moment at Midspan		
			Measured Strain $\times 10^{-6}$	Non-comp. = Strain / 1.0204	Mcomposite = Strain / 0.66243
Floor Beam 2					
Level 4/Line 1/ Pos. 2	388.1	291.1	187.1	183.4	282.4
Level 4/Line 2/ Pos. 2	392.3	294.2	188.5	184.7	284.6
Level 4/Line 3/ Pos. 2	780.4	585.3	375.2	367.7	566.4
Level 4/Line 4/ Pos. 2	296.7	222.5	145.5	142.6	219.7
Level 5/Line 1/ Pos. 2	448.4	336.3	227.2	222.7	343.0
Level 5/Line 2/ Pos. 2	456.9	342.7	219.5	215.1	331.4
Floor Beam 3					
Level 4/Line 1/ Pos. 3	388.1	291.1	168.8	165.4	254.8
Level 4/Line 2/ Pos. 3	392.3	294.2	181.4	177.8	273.8
Level 4/Line 3/ Pos. 3	780.4	585.3	345.1	338.2	521.0
Level 4/Line 4/ Pos. 3	296.7	222.5	147.1	144.2	222.0
Level 5/Line 1/ Pos. 3	448.4	336.3	208.8	204.6	315.2
Level 5/Line 2/ Pos. 3	456.9	342.7	211.7	207.0	319.6
Floor Beam 4					
Level 4/Line 1/ Pos. 4	388.1	291.1	177.9	174.3	268.6
Level 4/Line 2/ Pos. 4	392.3	294.2	168.4	165.0	254.2
Level 4/Line 3/ Pos. 4	780.4	585.3	358.0	350.8	540.4
Level 4/Line 4/ Pos. 4	296.7	222.5	136.1	133.4	205.5
Level 5/Line 1/ Pos. 4	448.4	336.3	215.8	211.5	325.8
Level 5/Line 2/ Pos. 4	456.9	342.7	195.6	191.7	295.3

Table 6a/ Member Forces on the West Truss Due to Trucks Loaded to 780 kN

Member	Line 3								Line 4							
	Pos. 1		Pos. 2		Pos. 3		Pos. 4		Pos. 1		Pos. 2		Pos. 3		Pos. 4	
	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.
Top Chord																
1-2	-746	-606	-662	-558	-527	-445	-390	-329	-565	-434	-501	-397	-400	-345	-295	-261
2-6	-936	-813	-982	-856	-786	-728	-583	-511	-709	-610	-746	-622	-595	-505	-442	-358
6-10	-988	-738	-1195	-894	-1283	-973	-1204	-911	-740	-547	-905	-650	-972	-703	-914	-659
Bottom Chord																
1-5	512	21	455	- 45	362	-129	269	-171	388	12	345	-56	274	-117	204	-131
5-9	1084	386	1172	433	1202	401	898	284	818	281	887	316	913	294	681	202
Web																
2-5	572	400	710	479	573	423	423	304	433	291	541	358	431	306	322	223
5-6	-200	-237	-256	-334	-561	-574	-425	-511	-147	-177	-189	-242	-427	-445	-322	-375
6-9	-126	-135	-42	-31	113	72	413	303	-98	-109	26	-17	83	60	313	233

Table 6b/ Member Forces on the West Truss Due to Trucks Loaded To 890 kN

Member	Line 1								Line 2							
	Pos. 1		Pos. 2		Pos. 3		Pos. 4		Pos. 1		Pos. 2		Pos. 3		Pos. 4	
	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.
Top Chord																
1-2	-270	-246	-241	-236	-191	-184	-142	-132	-588	-480	-521	-432	-416	-368	-310	-269
2-6	-339	-275	-353	-287	-285	-244	-211	-153	-734	-631	-775	-656	-618	-538	-461	-401
6-10	-363	-270	-440	-323	-465	-353	-436	-323	-764	-576	-934	-697	-1010	-759	-951	-709
Bottom Chord																
1-5	186	-26	165	-14	131	-54	98	-70	403	10	358	-54	286	-126	212	-141
5-9	403	152	425	169	434	174	325	127	838	309	917	341	947	316	699	212
Web																
2-5	208	149	253	181	206	165	154	125	448	324	562	390	450	333	336	242
5-6	-75	-83	-97	-123	-205	-205	-153	-177	-146	-164	-194	-237	-444	-458	-338	-377
6-9	-44	-29	20	10	42	56	148	133	-107	-90	21	-7	83	75	320	245

Table 7/ Member Forces On The West Truss Due to Trucks Loaded to 780 kN

with: 1) Expansion Bearing at One End Only
2) Fixed Bearings at Both Ends (in Parentheses)

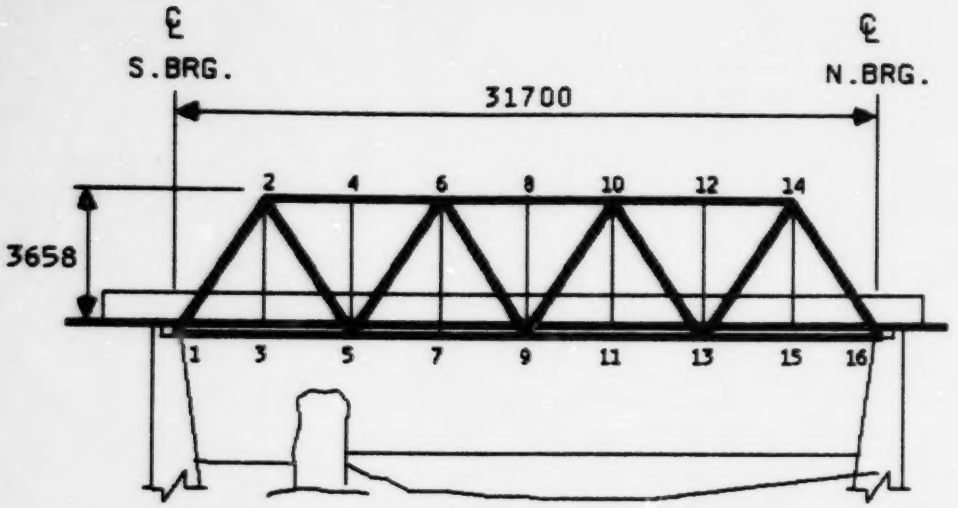
Member	Line 3, kN							
	Pos. 1		Pos. 2		Pos. 3		Pos. 4	
	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.
Top Chord								
1-2	-746 (-747)	-606	-662 (-663)	-558	-527 (-527)	-445	-390 (-392)	-329
2-6	-936 (-937)	-813	-982 (-987)	-856	-786 (-785)	-728	-583 (-583)	-511
6-10	-988 (-990)	-738	-1195 (-1205)	-894	-1283 (-1285)	-973	-1204 (-1211)	-911
Bottom Chord								
1-5	512 (-96)	21	455 (-217)	-45	362 (-376)	-129	269 (-402)	-171
5-9	1084 (480)	386	1172 (501)	433	1202 (469)	401	898 (225)	284
Web								
2-5	572 (572)	400	710 (717)	479	573 (570)	423	423 (424)	304
5-6	-200 (-205)	-237	-256 (-252)	-334	-561 (-570)	-574	-425 (-424)	-511
6-9	-126 (-134)	-135	-42 (44)	-31	113 (105)	72	413 (424)	303

- Compression
+ Tension

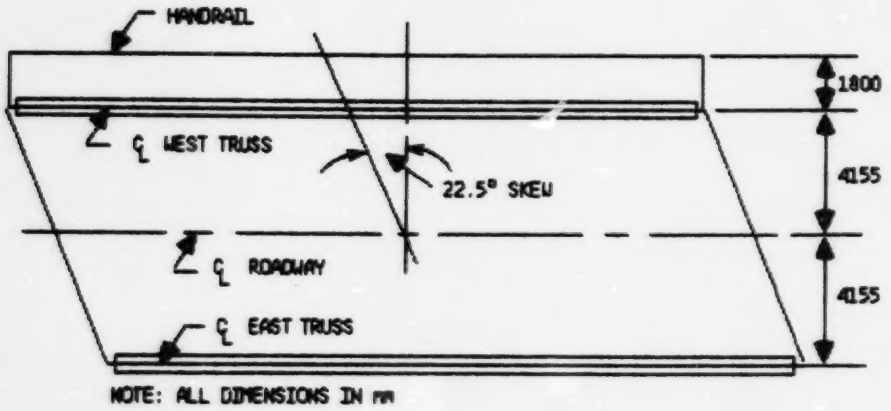
Table 8/ Measured and Calculated Deflection of Truss Under Two Trucks

Line 3	Vertical Displacement of Nodal Point 9, mm							
Load Level	Pos. 1		Pos. 2		Pos. 3		Pos. 4	
	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.
1	15.94	7.44	18.3	8.50	19.32	8.87	17.34	8.07
2	19.79	9.30	22.89	10.90	24.44	11.4	22.15	10.8
3	23.65	11.70	27.49	13.70	29.55	14.6	26.97	13.6
4	26.82 (19.82)	12.40	32.12 (22.18)	14.70	34.69 (22.83)	16.1	31.88 (18.52)	14.7

NOTE: The deflection values shown in parentheses are calculated only for a truss with fixed bearings at both ends. The rest of the calculated values are based on expansion bearings at one end and fixed at the other.



SECTION ALONG ℄ OF ROAD, WEST TRUSS



PLAN VIEW

Figure 1/ Bridge Configuration

MEMBER	MEMBER SIZES (IMPERIAL UNITS)
TOP CHORD	
1-2, 14-16	2-C12 x 25; 1-PL 18" x 3/8"
2-6, 10-14	2-C12 x 25; 1-PL 18" x 3/8"
6-10	2-C12 x 25; 1-PL 18" x 3/4"
BOTTOM CHORD	
1-5, 13-16	10WF54
5-13	10WF77
WEB	
6-7, 10-11	10WF66
2-3, 4-5, 12-13 14-15, 8-9, 2-5, 5-6	10WF45

TRUSS MEMBER DETAILS

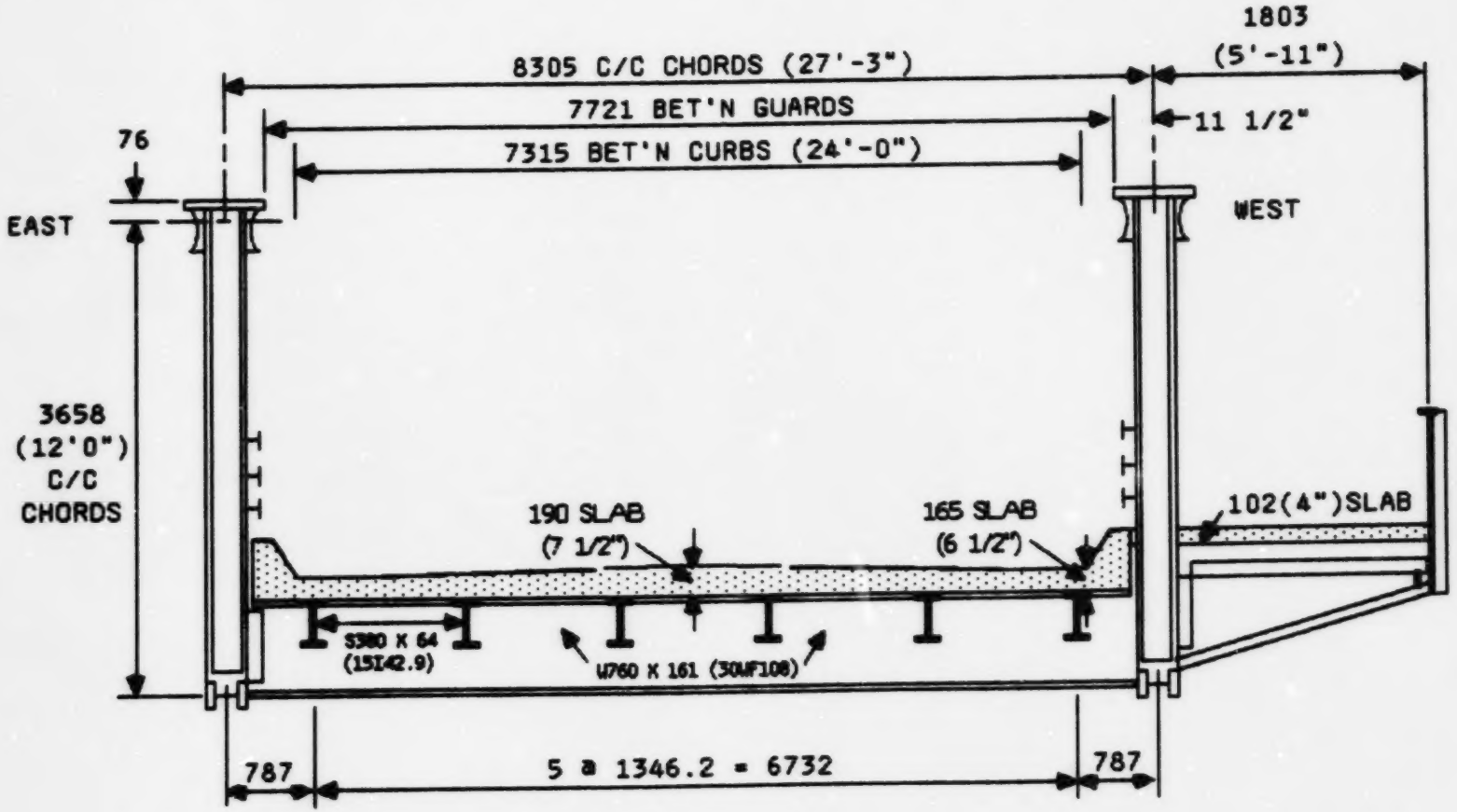


Figure 2/ Cross Section of the Bridge

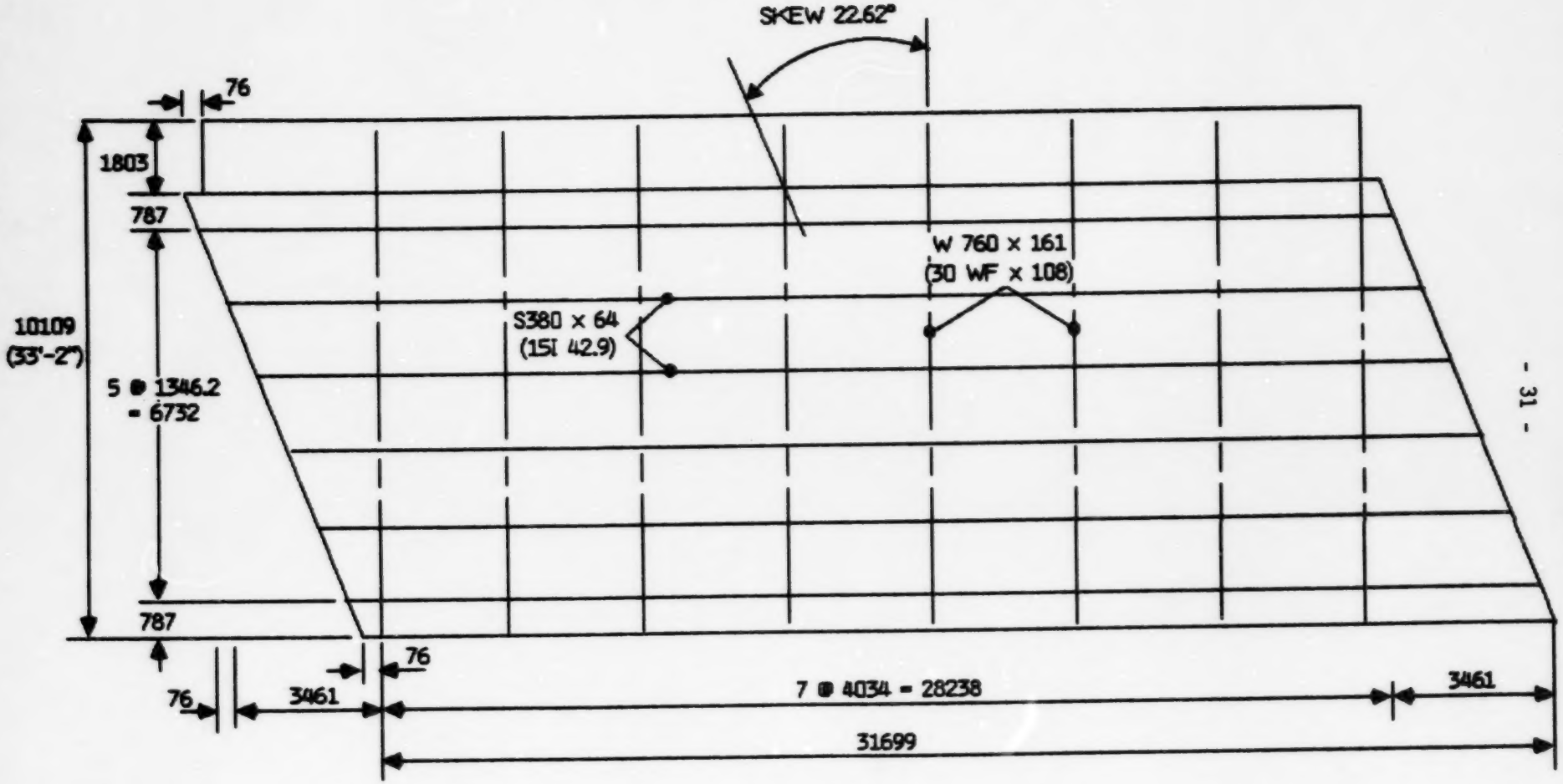
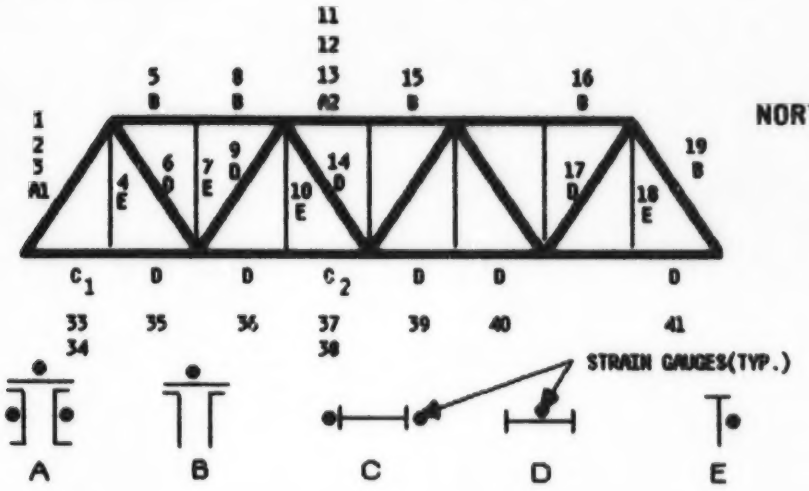
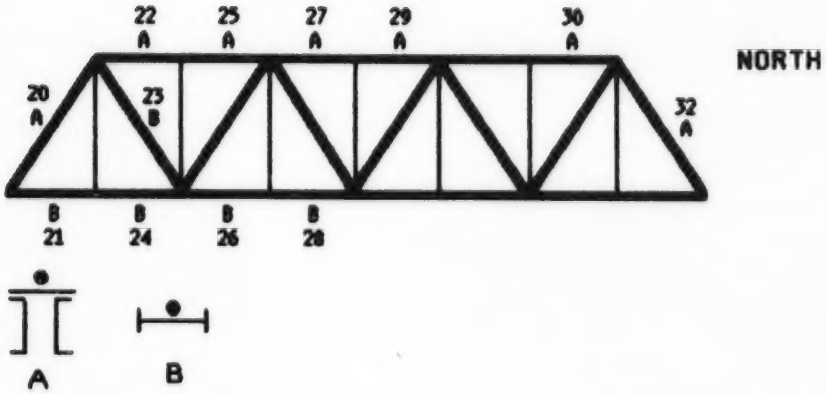


Figure 3/ Floor Plan of the Bridge



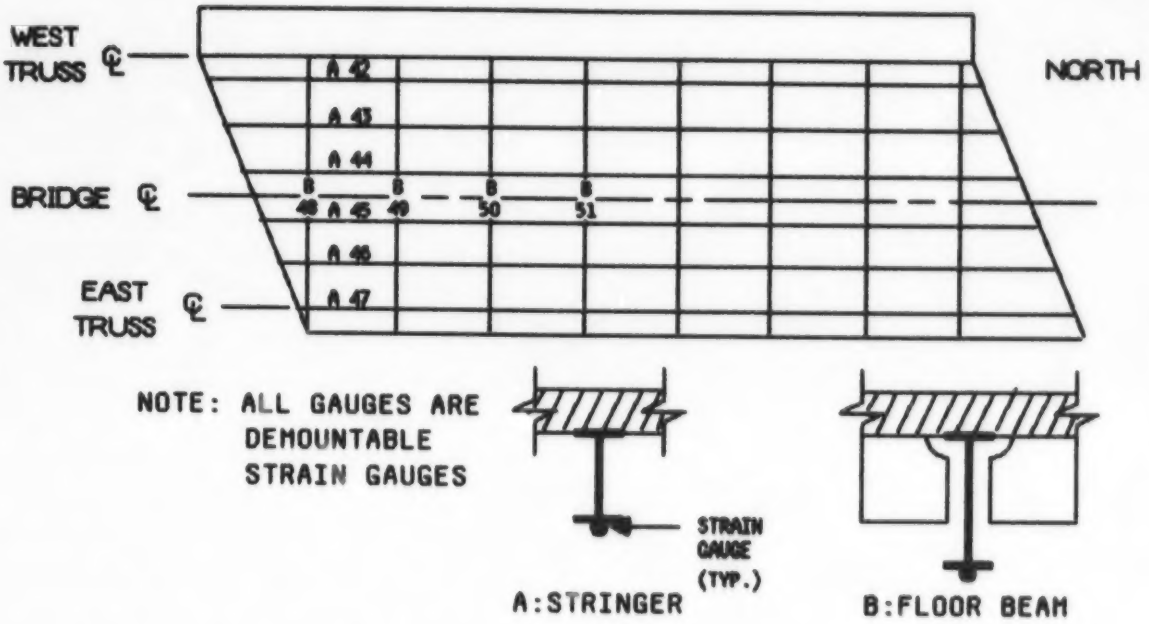
NOTE: BOTTOM CHORD GAUGES ARE DEMOUNTABLE STRAIN GAUGES

POSITION OF STRAIN GAUGES ON WEST TRUSS

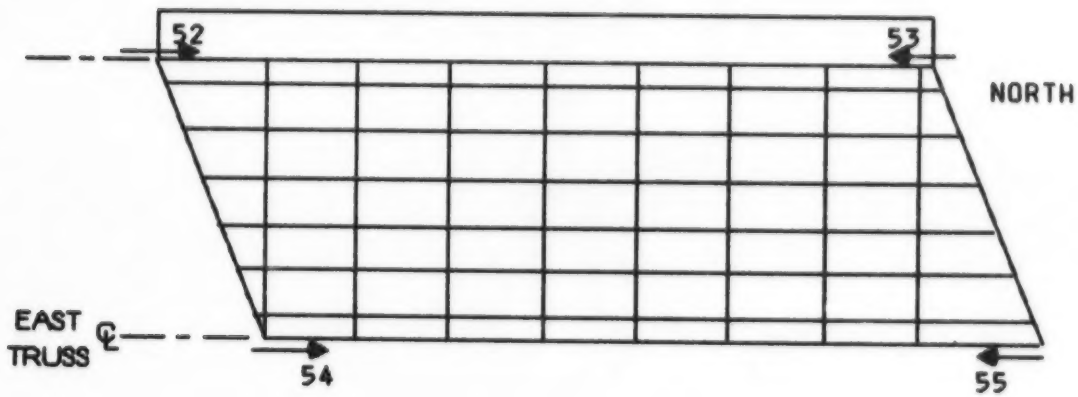


POSITION OF STRAIN GAUGES ON EAST TRUSS

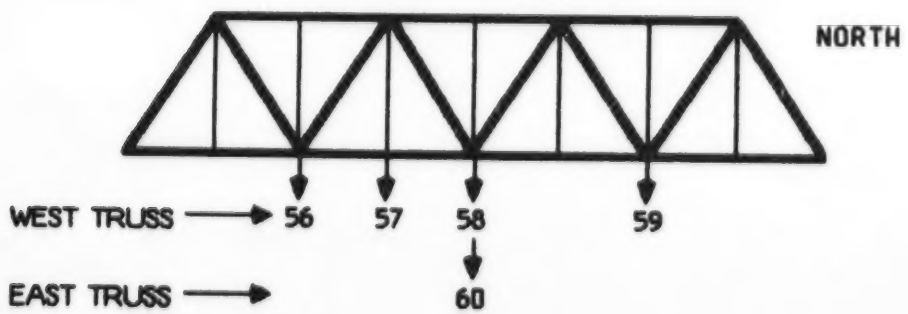
Figure 4/ Instrumentation of the Bridge



LOCATION OF STRAIN GAUGES ON STRINGERS AND FLOOR BEAMS

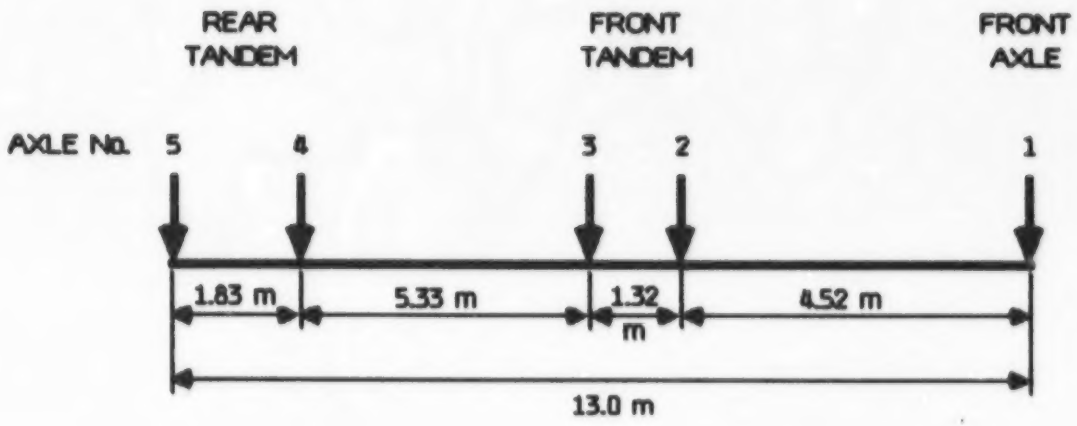


DISPLACEMENT TRANSDUCERS (HORIZONTAL)

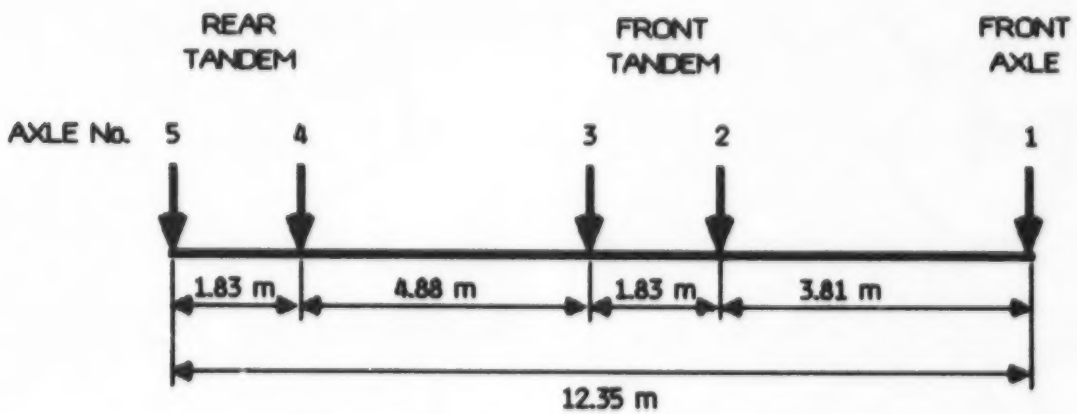


DISPLACEMENT TRANSDUCERS (VERTICAL)

Figure 5/ Instrumentation of the Bridge



AXLE CONFIGURATION OF TEST VEHICLE No. 1, "KENWORTH"



AXLE CONFIGURATION OF TEST VEHICLE No. 2, "MAC"

Figure 6/ Axle Configuration of Test Vehicles

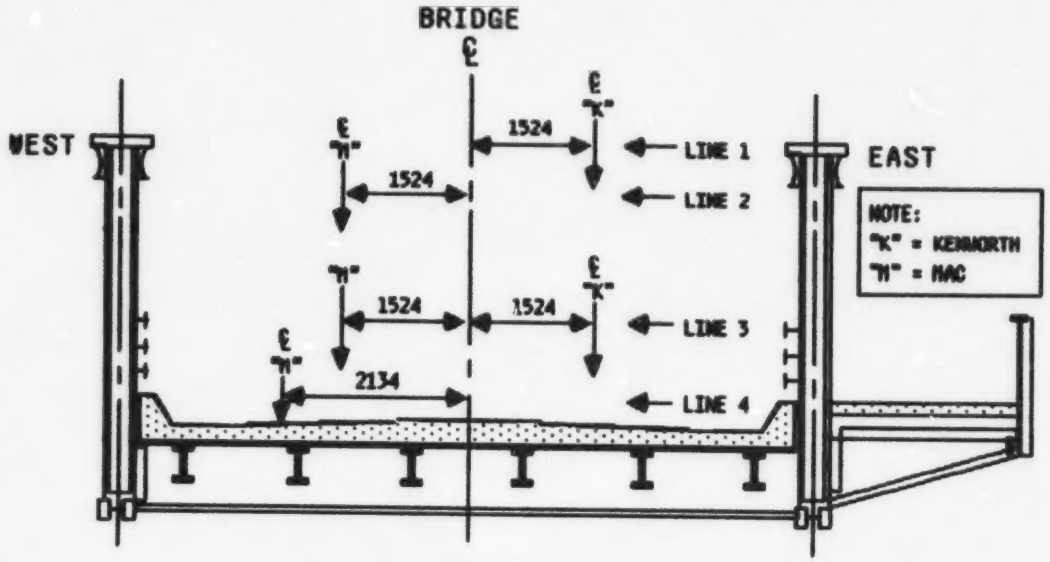


Figure 7a/ Transverse Locations (Lines) of the Test Trucks

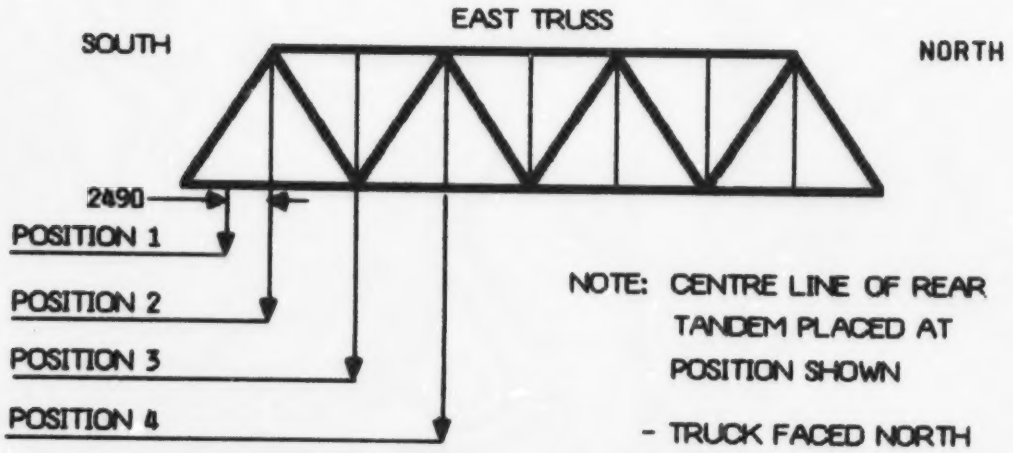


Figure 7b/ Longitudinal Positions of the Test Truck

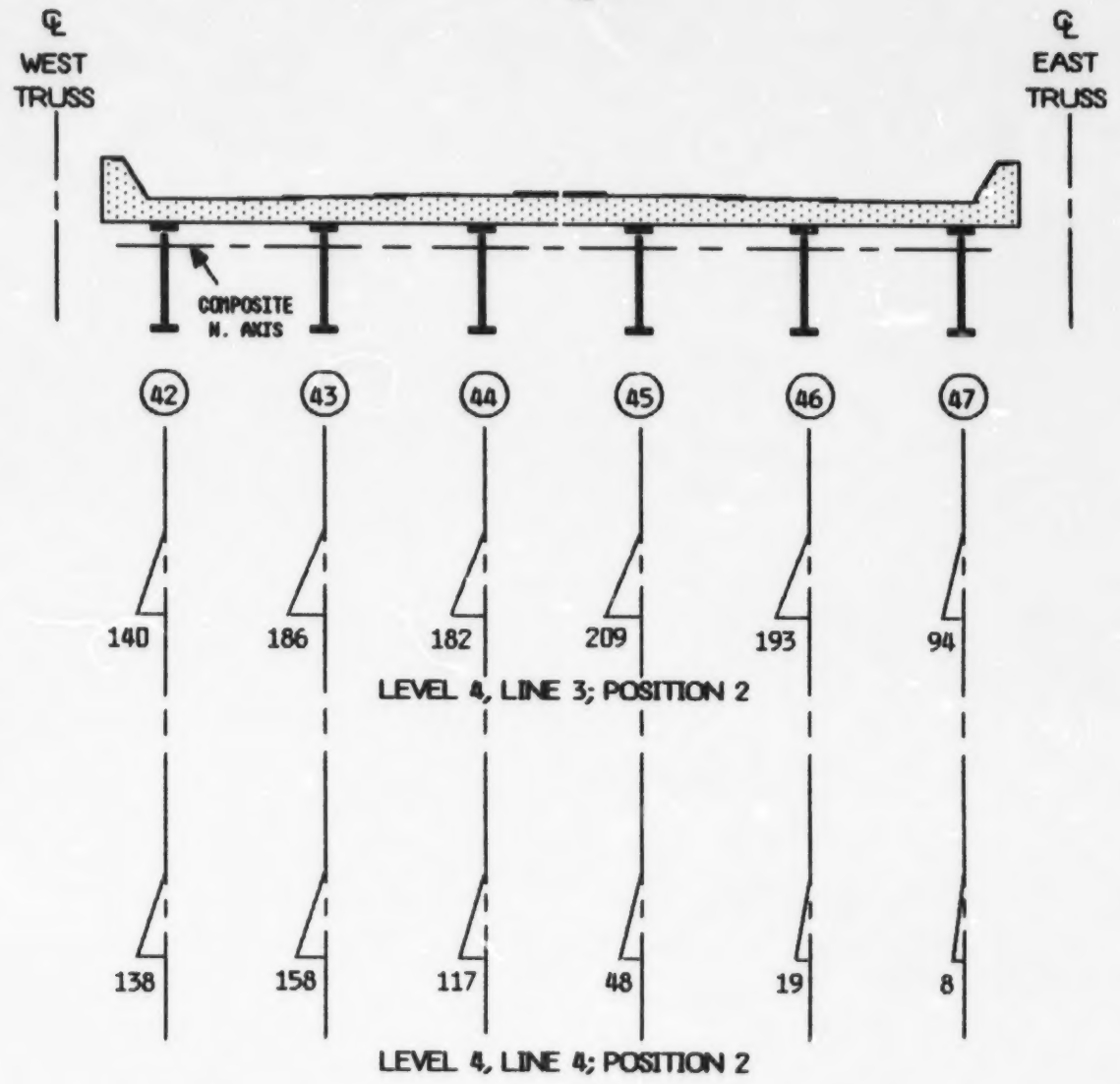


Figure 8/ Typical Strain Diagrams for Stringers

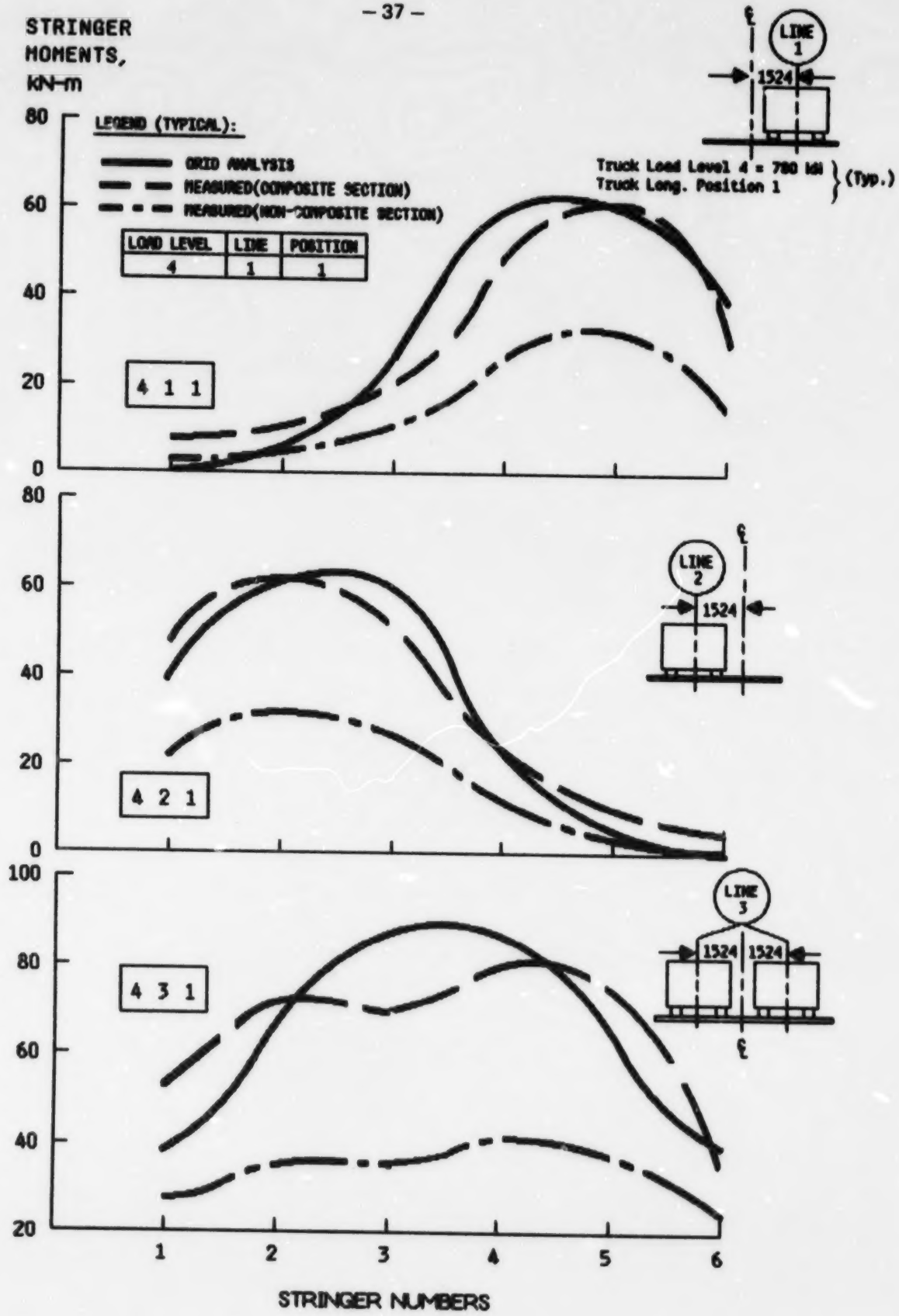


Figure 9/ Stringer Moments - Load Level 4

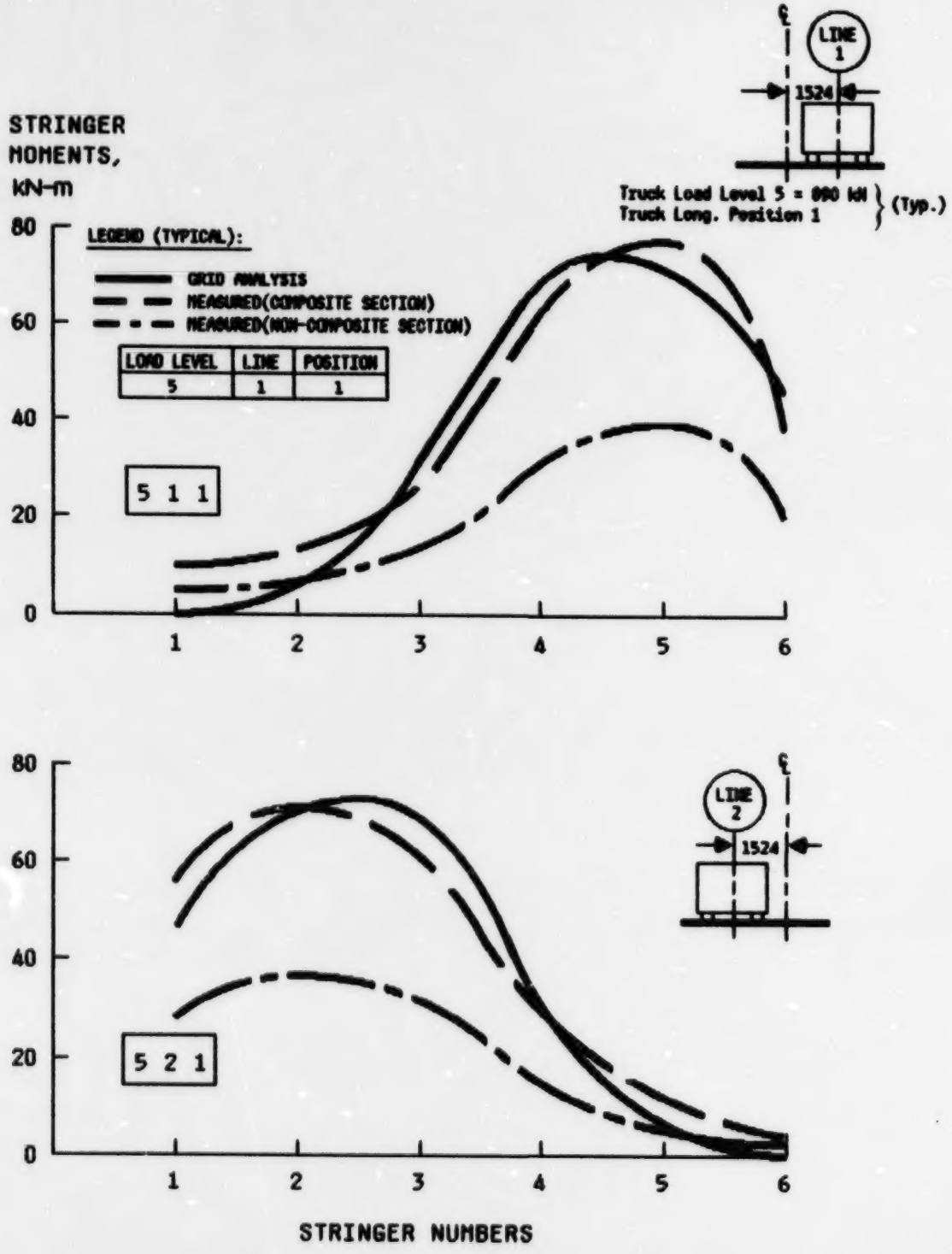


Figure 10/ Stringer Moments - Load Level 5

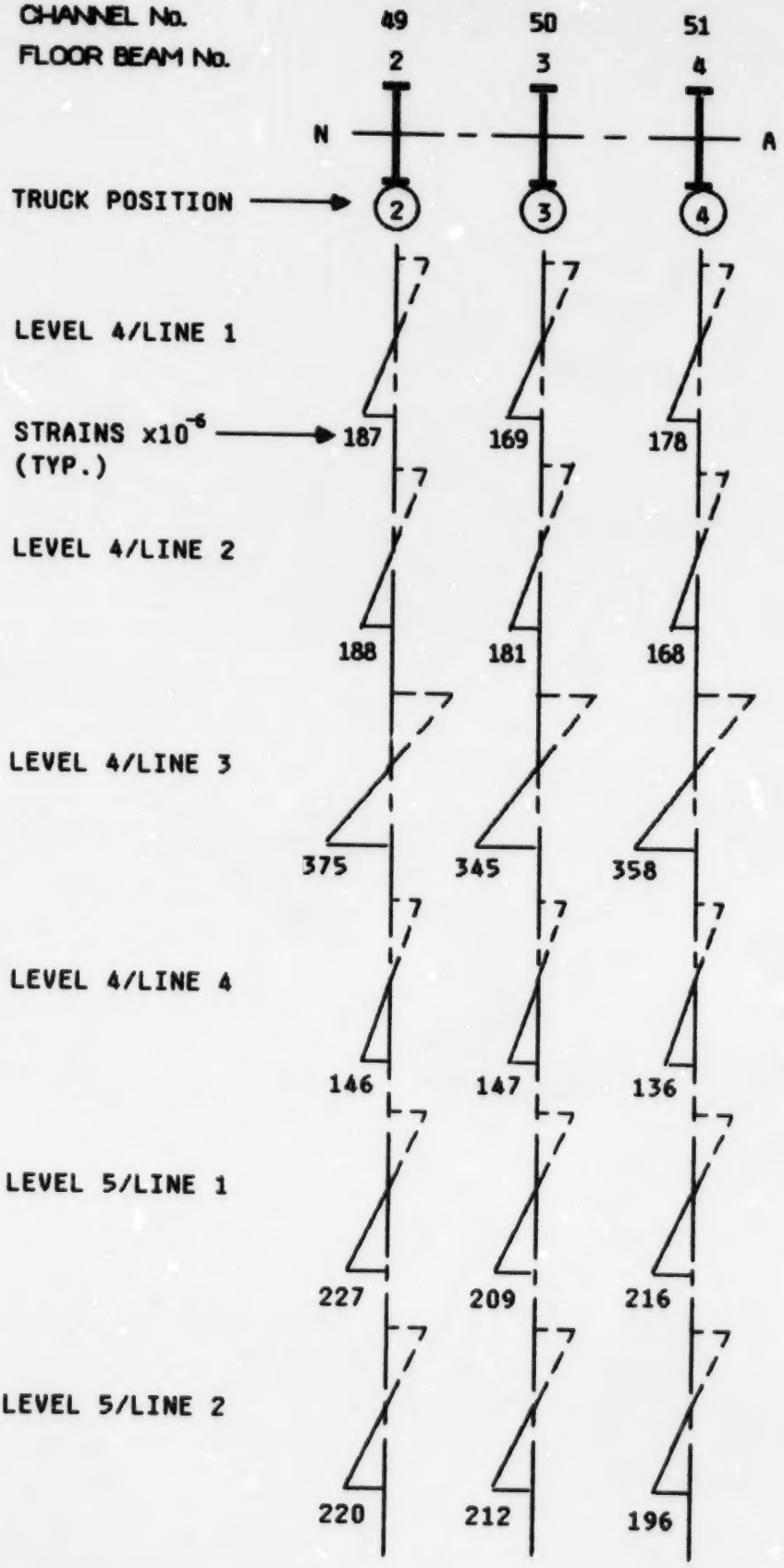


Figure 11/ Typical Strains Measured On the Floor Beams

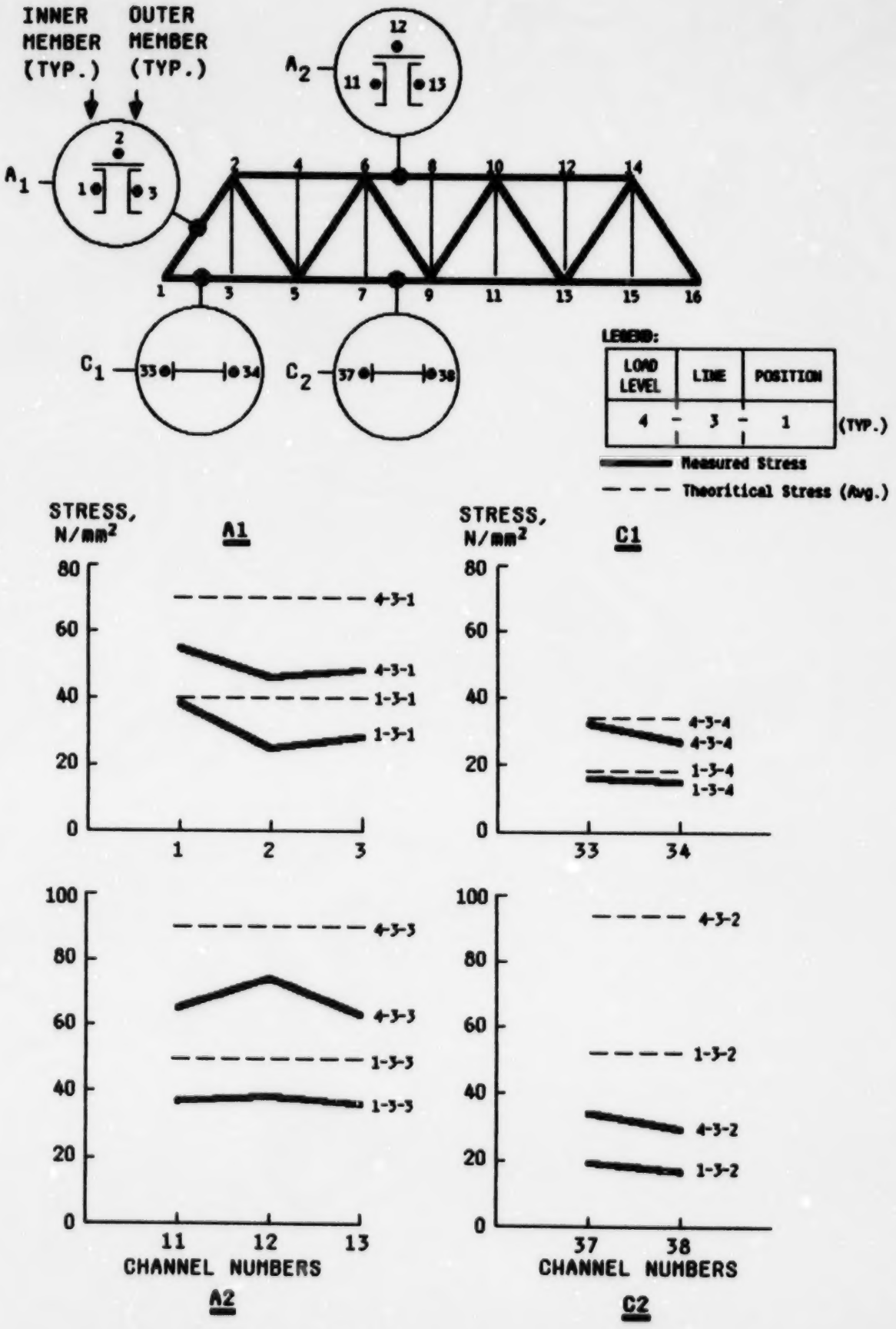


Figure 12/ Typical Stress Variations Observed in the Components of the Built Up Truss Members